DESIGN OF A REINFORCED CONCRETE GYMNASIUM BUILDING COMPOSED OF A SEMICIRCULAR ARCH AND BASEMENT.

A THESIS

Submitted for the degree of

MASTER OF SCIENCE IN CIVIL ENGINEERİNG.

By

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PREFACE

The purpose of this thesis is to give the design of a gymnasium building, built of a semi-circular arc and basement. It has been attempted to present the material in such a form that would be of most assistance to a student or unexperienced engineer his first design of such a building.

For information concerning the design of the arch, " Notes on the Design of Concrete Structures," by Prof. F.C. SNOW has been used. Books referred for information concerning the remaining designs are the following: " Reinforced Concrete Design," Sutherland and Clifford; " Handbook of Building Construction," Hool and Johnson, vol. 1. Stress values and code specifications have been obtained from the " Handbook of Reinforced Concrete Design."

Having already taken up the design of a " Rigid Frame Roof" and an "Open Spandril Arch Bridge" in class, I, upon the suggestion of prof. F.C. Snow, have chosen this type of construction so as to have a somewhat complete knowledge on the various types of arch structures.

The thesis was worked individually with the kind and willing assistance given by Prof. F.C. Snow, of the Georgia School of Technology, throughout the preparation of this thesis, to whom I wish to express sincere appreciation.

INDEX.

Pages.

Assembly drawings at the end of the thesis.

INTRODUCTION.

The thesis consists of the design of a Gymnasium building built of a semicircular arch having a span of 40 ft. and a basement. The main objective being the design of the arch. All primary items concerning the construction of the building have been designed to detail. Due to lack of time, architectural features, such as the layout of offices and utility rooms, andthe detailed design of doors and windows have not been taken up.

Throughout the thesis, a brief discussion of design procedures preceding each design, are given. Detailed drawings of each design have been placed along with their computations. Final assembly drawings are found at the end of the thesis.

THEORY OF CURVED BEAMS WITH VARIABLE MOMENT OF INERTIA

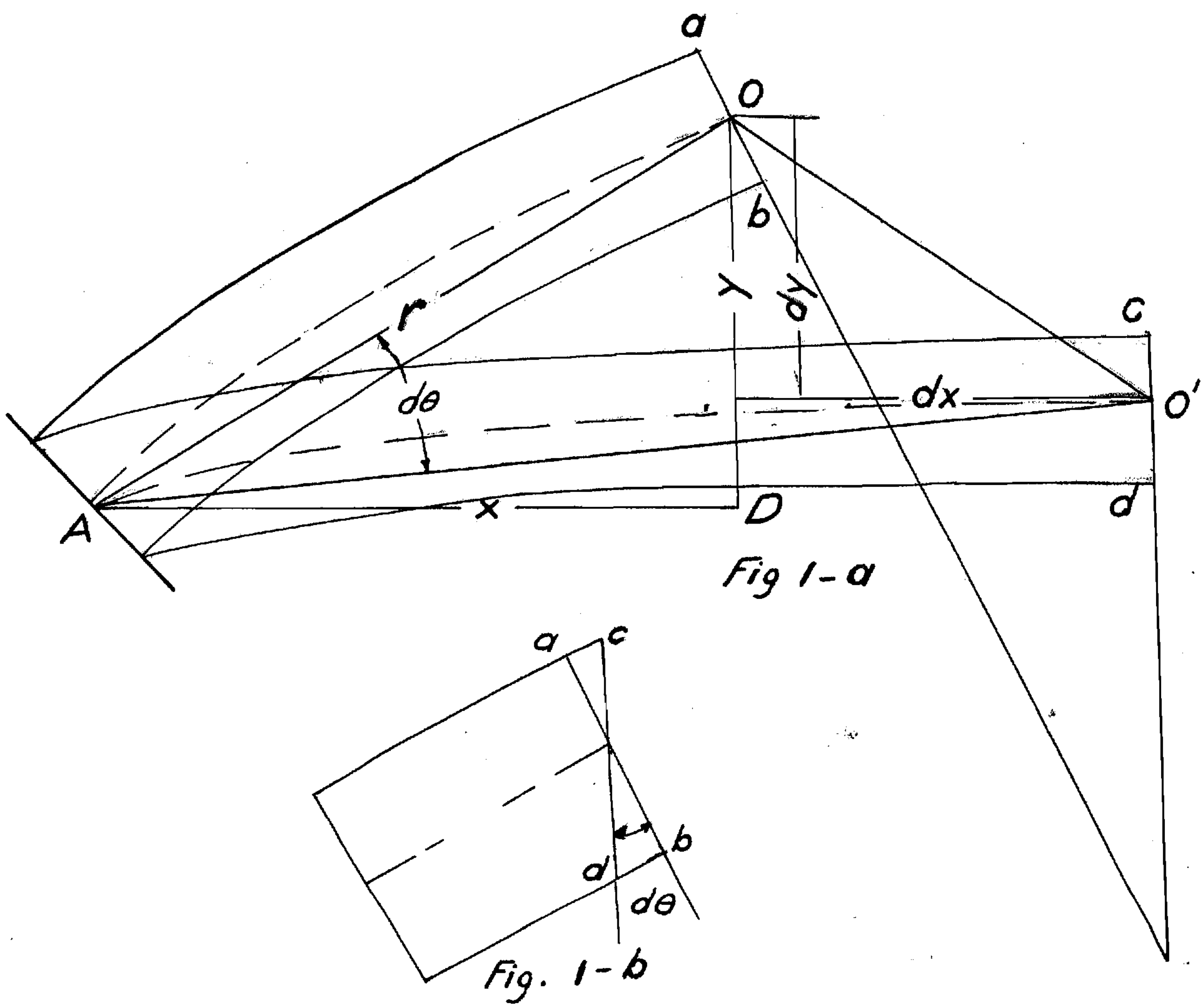


Fig. 1-b

A cross section of a curved rigid frame in which a is the left end and B (not shown) is the right end, has before bending, the position a-b and after bending the position c-d, turning through the angle dO, (See Fig. 1-a.)

First, assuming that a-b turns through dO to c-d without the point O moving. (See FIG. 1-b.) db = cdθ.

The unit stress in db=cdO, and unit stress per unit of length = cdOE/ds. E is the modulus of elasticity of the material.

But unit stress per unit of length = Mc/I, where I is the moment of inertia of the section so that:

$$\frac{Mc/I}{I} = \frac{cd\theta E}{ds} \quad \text{From which} \quad d\theta = \frac{Mds}{EI} \qquad (A)$$

M = moment at the section.

Second, taking into account the motion from O to O'(Fig. 1-a) triagle AOD is similar to triagle OO'C, so that:

$$\frac{dx}{rd\theta} = \frac{y}{r} \quad \text{or} \quad dx = yd\theta \qquad (B)$$

CURVED BEAMS DESIGN

Substituting the value of dO from (A) in (B) ,

$$dx = \frac{Myds}{EI} = \frac{My\Delta}{E} \quad \text{where } \Delta = \frac{ds}{I}$$

Since dx is the change in the span length (L) for the length of the frame ds, the total change for the entire span length (L) is,

$$Dx = \frac{My\Delta}{E}$$

<u>Temperature Changes</u>.

If Dx_t = change in span length due to changes of temperature, then,

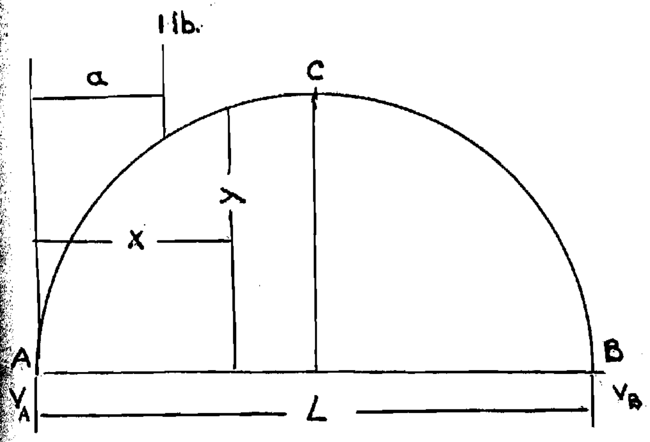
$$Dx_t = etLE \quad \text{where,}$$

e = coefficient of expansion of the material for 1 F.

t = the change of temperature in degrees F.

$L =$ the span in feet

$E =$ the modulus of elasticity of the material



In Fig. 2, the value of Dx at C for the left half A to C is -Dx for the right half C to B, or

$$\sum_A^C \frac{My\Delta}{E} = -\sum_C^B \frac{My\Delta}{E}$$

Since E is constant it can be cancelled out, and,

$$\sum_A^C My\Delta = -\sum_C^B My\Delta$$

or $$\sum_A^B My\Delta = 0$$

This is true if temperature is not concidered. Concidering temperature, $$\sum_A^B My\Delta + et^{LE} = 0$$

Vertical loading on a frame: See Fig. 2. If a unit load is placed at any point (a) distant from the left end (A) and the frame can rotate at (A), the moment at any point is:

for $x < a$

$$M = V_o x - H_o y$$

and

for $x > a$

$$M = V_o x - H_o y - (x-a)$$

multiplying both equations by $y\Delta$

$$\sum_A^a My\Delta = V_o \sum_A^a xy\Delta - H_o \sum_A^a y^2\Delta$$

$$\sum_a^B My\Delta = V_o \sum_a^B xy\Delta - H_o \,{}_a^B\, y^2\Delta - \sum_a^B (x-a)y\Delta$$

so that:

$$\sum_a^B My\Delta = V_o \sum_A^B xy\Delta - H_o \sum_A^B y^2\Delta - {}_A^B (x-a)y + etLE = 0$$

Let $V_o = 1 - \frac{a}{L}$ $\quad x = z\frac{dx}{2}$ $\quad a = K\frac{dx}{2}$ $\quad L = 20\frac{dx}{2}$ then

$$\frac{dx}{2}\sum_A^B xy - \frac{K}{20}\frac{dx}{2}\sum_A^B zy\Delta - H_o \sum_A^B y^2\Delta - \frac{dx}{2}\sum_a^B zy\Delta - \frac{dx}{2}\sum_a^B Ky\Delta$$

$$- etLE = 0$$

Let $\sum_A^B zy\Delta = \frac{20}{2}\sum_A^B y\Delta$ Since z for 3 - z for 3' = 20 etc.

Then:

$$\frac{dx}{2}\sum_A^B zy\Delta - \frac{dx}{2}\sum_a^B zy\Delta - \frac{Kdx}{4}\sum_A^B y\Delta - H_o\sum_A^B y^2\Delta + \frac{Kdx}{2}\sum_A^B y\Delta$$

$$-\frac{dx}{2}\sum_A^a Ky\Delta + etLE = 0$$

$$\frac{dx}{2}\sum_A^a zy\Delta - \frac{dx}{2}\sum_A^a Ky\Delta - \frac{Kdx}{4}\sum_A^B y\Delta - H_o\sum_A^B y^2\Delta - etLE = 0$$

$$\frac{\frac{K}{4}\sum_A^B y\Delta - \frac{1}{2}\sum_A^a (K-z)\, y\Delta}{\left(\sum_A^B y^2\Delta\right)\frac{1}{dx}} = H_o \quad (1)$$

Not concidering temperature

and for temperature changes,

$$H_t = \frac{etLE}{\sum_A^B y^2\Delta} = \frac{et(10dx)E}{\sum_A^B y^2\Delta} = \frac{(.000006)(10)(144)(2)(10^6 t}{\frac{1}{dx}\sum_A^B y^2\Delta}$$

$$H_t = \frac{17280T}{\frac{1}{dx}\sum_A^B y^2_\Delta} \quad (2)$$

Wind Loadings.

Suppose that the frame (Fig. 3-a) was part of a building and as such was subjected to wind pressures from the left of w lbs. per vertical Square feet from A to C.

$$V_o = \frac{wh^2}{2L} \qquad V = \frac{w}{2L}(h-y)^2$$

$$H = H_o - wy \quad \text{from A to C}$$

$$= H_o - wh \quad \text{from C to B}$$

Between A and C,

$$M = -H_o y + V_o x + \frac{wy^2}{2}$$

$$\sum_A^C My = -H_o \sum_A^C y^2\Delta + \frac{w}{2}\sum_A^C y^3\Delta$$

Between C and B,

$$M = -H_o y + V_o x + wh\left(y - \frac{1}{2}h\right)$$

$$\sum_C^B My = -H_o \sum_C^B y^2\Delta + V_o \sum_C^B xy\Delta + wh\sum_C^B y^2\Delta - \frac{wh^2}{2}\sum_C^B y\Delta$$

$\sum_C^B My\Delta + \sum_C^B My\Delta = 0$, so that,

$$H_o \sum_A^B y^2\Delta + V_o \sum_A^B xy\Delta + \frac{w}{2}\sum_A^C y^3\Delta + wh\sum_A^C y^2\Delta$$

$$-\frac{wh^2}{2}\sum_A^C y = 0$$

Substituting,

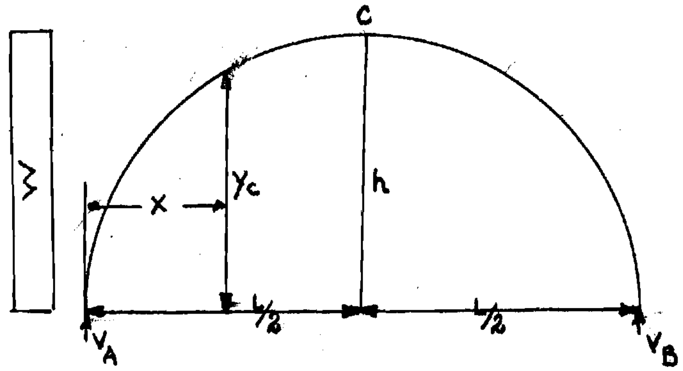
$$x = \frac{zdx}{2} \qquad \text{And} \qquad V_o = \frac{wh^2}{2L}$$

Likewise using,

$$\sum_C^B y = \frac{1}{2}\sum_A^B y \qquad H_o\left(\sum_A^B y^2\right) = \frac{wh}{2}$$

$$\sum_A^B xy\Delta = 10\sum_A^B y\Delta \qquad \text{and} \qquad dx = \frac{L}{10}$$

$$H_o = \frac{wh}{2} + \frac{\frac{w}{4}\sum_A^B y^3\Delta}{\sum_B^A y^2\Delta} \qquad (3)$$



Experiments with building models in wind tunnels indicate that if there is an opening between A and C there will be an internal pressure developed between C and B of wlbs. per Square feet acting to the right. For this value:

Between A and C,

$$M = -H_oy + V_ox$$

$$\sum_A^C My = -H_o\sum_A^C y^2_\Delta + V_o\sum_A^C xy\Delta$$

Between C and B,

$$M = -H_oy + V_ox - \frac{w}{2}(h-y)^2$$

$$\sum_C^B My\Delta = -H_o\sum_C^B y^2\Delta + V_o\sum_C^B xy\Delta - \frac{wh}{2}^2\sum_C^B y\Delta + wh\sum_C^B y^2\Delta - \frac{w}{2}\sum_C^B y^3_\Delta$$

Solving as in the previous case,

$$H_o = \frac{wh}{2} - \frac{\frac{w}{4}\sum_A^B y^3\Delta}{\sum_A^B y^2\Delta} \qquad (4)$$

Likewise,

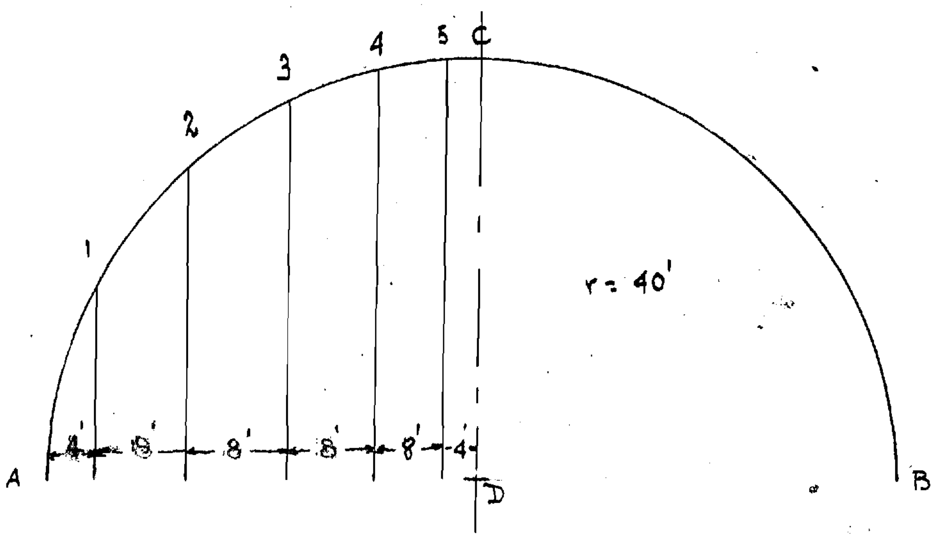
$$V_o = \frac{wh}{2L} \qquad V = \frac{w}{2L}(h-y)^2$$

$H_o = H$ between A and C

$H = H_o - w(h-y)$ between c and B

DESİGN OF THE ROOF FRAME

THE BUİLDİNG, is built of a semi-circular arc having a 80 ft. span and 40ft. rise. The roof is, $t_1 = 6$ in. thick, supported every 12 ft. by a rib constructed below the slab. The rib is assumed to be 16 in. wide, having a 2 ft. thickness at the crown and 4 ft. at the springing. The span is divided into 10-dx parts each 8 ft. long.



Assuming 4- 1 in. round bars at top and bottom:

Using 3000 lbs. concrete n = 10

$$nA_s = \frac{10 \times 8 \times .78}{144} = .434$$

Assumed values can be corrected after trial computations are made.

Points.	(1)	(2)	(3)	(4)	(5)	(6)	(7)
	X	y	t	ds	$(t-t_1)ds$	$150(t-t_1)ds$	t^3
1	4	17.50	3.45	16.00	47.00	7050	41.00
2	12	28.50	3.02	12.00	30.20	4530	27.50
3	20	34.75	2.72	9.40	20.90	3140	20.00
4	28	38.50	2.46	8.40	16.50	2480	14.80
5	36	40.00	2.12	6.00	9.70	1540	9.50

Points	(8)	(9)	(10)	(11)	(12)	(13)
	I_c	$\frac{t}{2}-d'$	$(\frac{t}{2}-d')^2$	$I_s=(\frac{t}{2}-d')nA_s$	$\frac{t}{2}$	$I = I_c + I_s$
1	4.14	1.555	2.41	1.050	1.725	5.190
2	2.88	1.340	1.80	0.780	1.510	3.660
3	2.05	1.190	1.42	0.615	1.360	2.665
4	1.50	1.060	1.12	0.485	1.230	1.985
5	0.96	0.890	0.78	0.338	1.060	1.298

Points.	(14)	(15)	(16)	(17)	(18)	(19)
	$\frac{ds}{I}=\Delta$	$y\Delta$	$y^2\Delta$	$y^3\Delta$	K or Z	$(K-Z)y\Delta$
1	3.47	60.7	1060	18600	1	0.00
2	3.83	109.0	3510	100000	3	60.70
3	3.78	131.0	4550	158000	5	169.00
4	4.40	169.0	6340	244000	7	200.70
5	6.17	247.0	9880	396000	9	369.70
Totals		716.7	25340	916600		740.80

Points.	(20)	(21)	(22)	(23)
	$\sum_A^a (K-z)\, y\Delta$	$\frac{1}{4} K\sum_A^a y\Delta$	col.21 - col.20	H_o
1	0.00	358.35	358.35	.0565
2	60.70	1075.05	1014.35	.1700
3	230.40	1791.75	1561.35	.2830
4	431.10	2508.45	2077.35	.3920
5	700.80	3225.15	2524.35	.5100
Totals	1423.00			

$$\frac{1}{4} \sum_A^B y\Delta = \frac{1}{2} \text{ (Sum of Col. 15)} = 358.35$$

$$\sum_A^B y^2\Delta = 2\text{(Sum of Col. 16)} = 50680$$

$$\sum_A^B y^3\Delta = 2 \text{ (Sum of Col. 17)} = 1833200$$

$$C_h = \frac{1}{dx} \sum_A^B y^2 = \frac{2}{dx}\text{(Sum of Col. 16)} = \frac{2}{8}(25340) = 6335$$

$H_o = \dfrac{\text{Val.in Col. 22}}{C_h}$ See formula 1 page

$H_t = \dfrac{(t)\ 17280}{C_h} = \dfrac{(t)\ 17280}{6335} = 2.73\ t$ See form. 2 page

For $t = 25^\circ$ F H_t = <u>68.2 lbs.</u>

For $t = -35^\circ$ F H_t = <u>-95.5 lbs.</u>

Computations for D.L. and L.L. Moment Coefficients

Points.	(24)	(25)	(26)	(27)	(28)	(29)
	K or z	V_o	H_o	m_3	$H_o y^3$	M_3
1	1	.95	.0565	3.00	1.96	1.04
2	3	.85	.1700	9.00	5.90	3.10
3	5	.75	.2830	15.00	10.30	4.70
4	7	.65	.3920	13.00	13.60	-0.60
5	9	.55	.5100	11.oo	17.70	-6.70
5'	11	.45	.5100	9.00	17.70	-8.70
4'	13	.35	.3920	7.00	13.60	-6.60
3'	15	.25	.2830	5.00	10.30	-5.30
2'	17	.15	.1700	3.00	5.90	-2.90
1'	19	.05	.0565	1.00	1.96	-0.96
Totals		5.00	2.8230			-22.92

For z less than K:

$M = V_O z(\frac{dx}{2}) - H_O y = m - H_O y =$ Col.27-Col.28 or Col.30-Col.31

For x greater than K:

$M = K(1 - \frac{z}{20})(\frac{dx}{2}) - H_O y = m - H_O y =$ Same as above(diff. of Col)

At point 3 : (z = 5)

$$z \frac{dx}{2} = 20 \qquad y = 34.75$$

$$(1 - \frac{z}{20})(\frac{dx}{2}) = 3.00$$

Points.	(30)	(31)	(32)
	m_5 1/2	H_oy_5 1/2	M_5 1/2
1	2	2.26	-0.26
2	6	6.80	-0.80
3	10	11.70	-1.70
4	14	15.70	-1.70
5	18	20.40	-2.40
5'	18	20.40	-2.40
4'	14	15.70	-1.70
3'	10	11.70	-1.70
2'	6	6.80	-0.80
1'	2	2.26	-0.26
Totals			-13.72

At point 5 1/2 (z=10)

$$z \frac{dx}{2} = 40$$

$$(1 - \frac{z}{20})(\frac{dx}{2}) = 2$$

Computations for Rib Weight Moments.

$V_3 = V_o$ - (Sum of loads at 1 and 2) = 18611 - (7050 - 4530)=7031

$V_{5\ 1/2} = 0 \qquad H_3 = H_{5\ 1/2} = H_o = 7580$

Temperature :

$H_t = H_3 = H_{5\ 1/2} = 68.2$ or -95.5

$M_t = -H_t y$

$$\begin{aligned} M_t &= +95.5 \times 34.75 = +3310 \\ &= -68.2 \times 40.00 = -2400 \end{aligned}$$ at point 3

$M_t = -2728$ or 3820 at point 5 1/2

Above numerical values are obtained from the next page.

Points.	(33)	(34)	(35)	(36)	(37)
	Rib wt.	V_o	H_o	M_3	$M_{5\frac{1}{2}}$
1	7050	6700	390	7330	-1830
2	4530	3840	770	14000	-3620
3	3140	2360	920	14800	-5340
4	2480	1610	970	-1490	-4200
5	1450	797	740	-9700	-3480
5'	1450	652	740	-12600	-3480
4'	2480	867	970	-16400	-4200
3'	3140	753	920	-16600	-5340
2'	4530	680	770	-13200	-3620
1'	7050	352	390	- 6850	-1830
Totals.		18611	7580	-39710	-36940
Temperature.			68.2	-2400	-2728
			95.5	3310	3820

D.L.

6 in. roof = 75 lbs. per $ft.^2$

D.L. = 75 x 12 = 900 lbs./ linear ft.

L.L.

30 $lbs./ft.^2$

L.L. = 30 x 12 = 360 lbs./ linear ft.

D.L. + L.L. = 900 + 360 = 1260lbs./ linear ft.

Ribs are 12 ft. center to center.

The 30 lbs. L.L. value is the effect of the wind on the structure.

D.L. plus L.L. Moment Computations.

Points.	(38)	(39)	(40)	(41)	(42)
	2000 ds	V_o	H_o	M_3	$M_{5\frac{1}{2}}$
1	20200	19200	1140	21000	-5250
2	151oo	12800	2560	46800	-12100
3	11800	8850	3340	55500	-20000
4	10600	6900	4150	-6350	-18000
5	7550	4260	3850	-50500	-18100
5'	7550	3490	3850	-65500	-18100
4'	10600	3700	4150	-70000	-18000
3'	11800	2940	3340	-62500	-20000
2'	15100	2260	2560	-43800	-12100
1'	20200	1010	1140	-19400	-5250
Totals.		65410	30080	-194750	-146900

$V_3 = 65410 - (20200 - 15100) = 30110$ $\qquad V_{5\frac{1}{2}} = 0$

$H_3 = H_{5\frac{1}{2}} = 30080$

Wind Loads.

Wind pressure = 30 lbs. per Sq.ft. $\qquad h = y_c = 40$ ft.

$W = W_1 = 30 \times 12 = 360$ lbs. per linear ft. of rib.

$$H_o = \frac{\frac{wh}{2} \pm \frac{w}{4} \sum_A^B y^3\Delta}{\sum_A^B y^2\Delta} \qquad V_o = \frac{wh^2}{2L}$$

$$H_o = 7200 \quad \frac{90(\text{Sum of col. 17})\ 2}{(\text{Sum of col. 16})\ 2} = 7200 \quad \frac{90(916600)}{25340} =$$

$$= 7200 \quad 3260 = 10460 \quad \text{or} \quad 3940$$

$$V_o = \frac{360(40)^2}{2L} = \frac{360(40)^2}{160} = 3600$$

H_o = 10460 when the wind acts against the left wall (left side)

H_o = 3940 when the wind acts on the inside of the right wall

Wind Loads on A.C. Acting to Right.

$H_o = 10640 \qquad H_3 = H_o - wy = 10460 - 360(34.75) = -2040$

$V_o = 3600 \qquad V_3 = \frac{w}{2L}(h-y)^2 = \frac{360}{160}(40-34.75)^2 = 61 = V_{3'}$

$$M3 = -H_oy_3 + V_ox_3 + wy_3^2 = -1460x34.75 + 3600x20 + \frac{360x34.75^2}{2} =$$

$$= -36200 + 72000 + \frac{435000}{2} = -72500$$

$$M_{3'} = -H_oy_{3'} + V_ox_{3'} + wh(y_{3'} - \frac{h}{2}) = -36200 + 216000 + 360x40(14.75) =$$

$$= 66000$$

$$H_{3'} = H_o - wh = 10460 - 360x40 = -3940$$

$$M_{5\frac{1}{2}} = -10640x40 + 3600x40 + 180x40^2 = 13600$$

$M_{5\frac{1}{2}}'$ = Same point as $M_{5\frac{1}{2}}$

$V_{5\frac{1}{2}} = 0$

Wind Loads C. to B. acting to Right.

$H_o = 3940 \qquad V_o = 3600 \qquad V_{5\frac{1}{2}} = 0 \qquad V_3 = V_{3'} = 61$

$M_3 = -3940x34.75 + 3600x20 + 180x34.75^2 = 152500$

$M_{3'} = -3940x34.75 - 3600x60 - 360x40x14.75 = 579000$

$H_{3'} = 3940 - 360x40 = -10460$

$M_{5\frac{1}{2}} = -3940x40 + 3600x60 + 180x40^2 = 274400$

$H_{5\frac{1}{2}} = 3940 - 360x40 = -10460$

Summary of Wind Loadings. (actingto right)

M_3	-72500	152500	80000
H_3	-2040	-8560	-10600
V_3	61	61	122
$M_{3'}$	-66000	579000	645000
$H_{3'}$	-3940	-10460	-14400
$V_{3'}$	61	61	122
$M_{5\frac{1}{2}}$	13600	274400	288000
$M_{5\frac{1}{2}}$	3940	-10460	-14400
$V_{5\frac{1}{2}}$	0	0	0

At point 3' the largest moment is 645000 if the wind acts to the left this will become -645000 acting at point 3 with corresponding $H_3 = -14400$ and $V_3 = 122$

Maximum Values of M, H, V.

At point 3 :

	M	H	V
Rid wt.	-39710	7580	7031
D.L. - L.L.	-194750	30080	30110
Temperature	-2400	68.2	0
Wind	-645000	-14400	122
Totals	-881860	23328.2	37263

AT point $5\frac{1}{2}$:

	M	H	V
Rib wt.	-36940	7580	0
D.L. - L.L.	-146900	30080	0
Temperature	-2728	68.2	0
Wind	-288000	14400	0
Totals	-474568	52128.2	0

If the wind acts from the left $M_{5\frac{1}{2}} = -288000$ and correspondly $H_{5\frac{1}{2}} = 14400$, Since these values will give the larger moment they are used.

Stresses Resulting from Maximum Moments.

At point 3 :

$\text{Cos. } \theta = .86$ $\qquad n = 10$

$\text{Sin. } \theta = .51$ $\qquad f_{c'} = 3000$

$H \cos.\theta = 23328.2 \times .86 = 20000$

$V \sin.\theta = 37263 \times .51 = \underline{19000}$

$39000 = N$

$p = \dfrac{8(.78)}{32.6(16)} = .012 \qquad n = 10 \qquad np = .12$

$x_o = \dfrac{M}{N} = \dfrac{881860}{39000} = 22.6 \qquad \dfrac{t}{x_o} = \dfrac{2.72}{22.6} = .12$

$\dfrac{d'}{t} = \dfrac{2}{32.6} = .055 \qquad \dfrac{Nx_o}{f_c b t^2} = .5$ (from diagram A)

Therefore : $f_c = \dfrac{881860(12)}{.5(16)(1060)} = 12500$ lbs. per in.^{2} (OK)

Allowable $f_c = .40 \times 30000 = 12000$ lbs./in.^{2}

(5 per cent variation allowable)

$u = \dfrac{37263}{22(.875)(2.58)(12)} = 63$ lbs./in.^{2} (O.K.)

Allowable u (bond) plain bars, ordinary anchorage. $.03 \times 30000 = 90$ lbs./in.^{2}

$v = \dfrac{37263}{16(.875)(31)} = 86$ lbs./in.^{2} (O.K.)

Allowable v (shear) with web reinforcement, ordinary anch. $.06 \times 30000 = 180$ lbs./in.^{2}

At point $5\frac{1}{2}$:

$H \cos.\theta = 52128.2 \times 1 = 52128.2$

$V \sin.\theta = 0$ (Angle $\theta = 0^\circ$) and $V_{5\frac{1}{2}} = 0$

Computing for the value of f_c using the same assumptions used in computing the f_c value at point 3, results in a f_c value of 4400 lbs./in.^{2} which is too big.

Assuming 8(1in.) round bars top and bottom :

$$p = \frac{16(.78)}{24(16)} = .0326 \qquad n = 10 \qquad np = .326$$

$$x_o = \frac{M}{N} = \frac{474568}{52128.2} = 9.1 \qquad \frac{t}{x_o} = \frac{2}{9.1} = .22$$

$$\frac{d'}{t} = \frac{2}{24} = .835 \qquad \frac{Nx_o}{f_c b t^2} = .25 \quad \text{(from diagram B)}$$

Therefore :

$$f_c = \frac{47456(12)}{.25(16)(576)} = 2460 \text{ lds./in.}^2$$

Though the value exceeds the allowable stress, no changes will be done. It is obvious that an addition of 6 ins. to the crown thickness would result in an f_c within the allowable limit.

In case such a change were to be done in the design the only changing value would be the rib weight moment.

Dimentions Concerning the Roof Frame.

Use 11 ribs 10 ft. c. to c. with a crown thickness of 2 ft. a springing thickness of 4 ft. and a width of 16 inches.

Between points 3 and 3' use 8(1 in. round) bars top and bottom. Between points o and 3, o' and 3', use 4(1 in. round) bars top and bottom.

Since this is an eccentric column design the top steel has to be tied to bottom steel with $\frac{1}{4}$" round ties about 8" apart.

Dimentions concerning the relative values of t are shown on drawing showing the crossectional views of the ribs.

The ribs are spaced 10 ft. apart, to have a convenient arrangement of floor beams and girders as will be seen later in the Thesis. It also brings down the f_c value at point $5\frac{1}{2}$.

DESİGN PROCEDURE FOR SLABS.

Since a slab, for design purposes, may be considered a rectangular beam, the stresses and resulting dimensions are computed as follows :

M = bending moment in inch-lbs.

W = total uniform load in either direction, on a 1-ft. width of slab in lbs.

L = clear span in inches. b = 12 inches

d = effective depth; distance from extreme fobers in compression to center of gravity of tensile reinforsement in inches.

j = ratio of distance between center of compression of concrete and center of tension steel to effective depth of slab.

f_s = tensile unit stress in the steel in lbs. per in.^{2}

A_s = area of tension steel required in a 1 ft. width of slab in in.^{2}

Then :

M = WL/10 for semi-continuous spans.

M = WL/12 for fully continuous spans.

$$A_s = \frac{M}{f_s jd} \qquad d = \sqrt{\frac{M}{Rb}}$$

In in working out the required value of d, the value of M is the greater of the two bending moments. After the slab thickness is determined by adding the required fire-proofing allowance to the depth d, the weight of the slab as originally assumed is checked and the slab redesigned if any appreciable difference exists.

The shear is not usually a determining factor in slab design exept for heavily loaded slabs of short span. The same formulas, apply, however for rectangular beams, the entire side of the beam being used for the size b, and for V, one half the total load carried in the direction considered.

On the other hand, BOND-STRESSES are often high in typical slab designs, and should be determined by the same formulas used for rectangular beams, the computations being made for a 1-ft. width of slab. Adequate ANCHORAGE should be provided.

In order to provide for the negative bending moment on each support in contineous slabs, which in are analogous to rectangular (in principle) beams, a portion of the main reinforcement is raised at about the fifth point of the clear span, and carried accross the beam 1-in. below the top of the slab. The number of the bars should be so determined that the sectional area of the steel over a support satiesfies the requirement of the design moment at that section. In the case of slightly restrained end spans, every other rod, 50 per cent, of the steel adjacent to the supports should be raised. In general the bars should be bent at 30° to the horizontal.

DESİGN PROCEDURE FOR RECTANGULAR BEAMS.

(1) The width of, b, of the beam, or the girder, is made large enough to accomodate the probable reinforcement, including the necessary protection; the weight of the beam is then computed on the basis of the assumed width and a depth d,

(2) The maximum shear is determined.

(3) The maximum bending moments at the supports and near the mid-span and under the concentrations, for beams subjected to concentrated loads.

(4) The depth is computed by formula $d = \sqrt{\frac{M}{Rb}}$

(5) Area of main reinf. steel by formula $A_s = \frac{M}{f_s\ jd}$

(6) Value of max. unit shearing stress $v = \frac{V}{bjd}$

(7) The bond stress and anchorage are tested $u = \frac{V}{\Sigma o jd}$

DESİGN PROCEDURE FOR RECTANGULAR GİRDERS.

The process of designing GIRDERS is identical to the process of designing RECTANGULAR BEAMS, the only difference being that the moments are computed for concentrated loads, coming from the beam reactions.

For a slab on which the entire load is considered to be carried on the steel spanning the shorter dimension, additional reinforcement, consisting of 3/8 in. round rods, spaced 1 ft. 6 inches on centers, or an equivalent steel area, should be placed transversely to the main reinforcement. This steel, usually 3 or 4 rods per panel, serves to provide against shrinkage cracks and temperature cracks, and should be placed above the transverse reinforcement.

SLAB DESIGN

Data Sheet , for Slab - Beam - Girder Floor :

Live Load = 100 lbs./ft.^{2}

Floors : Wood

Materials :

Steel : Structural grade billet steel

Concrete : Max. size aggregate - 1 in.

Ultimate strength at 28 days 3000 lbs.

Spesifications : 1928 Joint Standard Building Code

Stresses lbs./in.^{2}:

$f_s = 18000$

$f_c = .40 \times 3000 = 1200$ (bending)

$= .45 \times 3000 = 1350$ (at supports)

$v = 60$ (without web reinf.)

$= 180$ (with web reinf.)

$u = 120$ (plain bars ordinary anch.)

Computations for Floor Slab

Allowable: $f_c = 1200$ $f_s = 18000$ $n = 10$ $v = 60$ $u = 120$

Clear span = 9'- 0" Assume 12" beams Slab 5" = 75

Bearing span = 10'- 0" Live Load = 100

Panel width Floors = 15

Total in lbs./ft.^{2} 190

$$M = \frac{wL^2}{12} = \frac{200 \times 9^2}{12} = 1350 \text{ ft.lbs./ft.strip} \quad \text{(Interior span)}$$

$$V = \frac{wL}{2} = \frac{190 \times 9}{2} = 855 \text{ lbs.}$$

Thickness:

$$d = \sqrt{\frac{M}{bR}} = \sqrt{\frac{1350 \times 12}{12 \times 108}} = 3.54''$$

$$\frac{+\ 1.25''}{4.75''} \text{ necessary depth}$$

USE 5" slab with d = 3.75"

Floor Slab : Steel

Interior span : $A_s = \frac{M}{f_s jd} = \frac{1350}{18000 \times 7/8 \times 3\ 3/4} = .0229 \text{in.}^2/\text{in.}$

For $\frac{1}{2}$ in. rounds Allowable spacing $= \frac{.1963}{.0229} = 8.6''$

USE 1/2 in. rounds at 8" c. to c. (Int. steel)

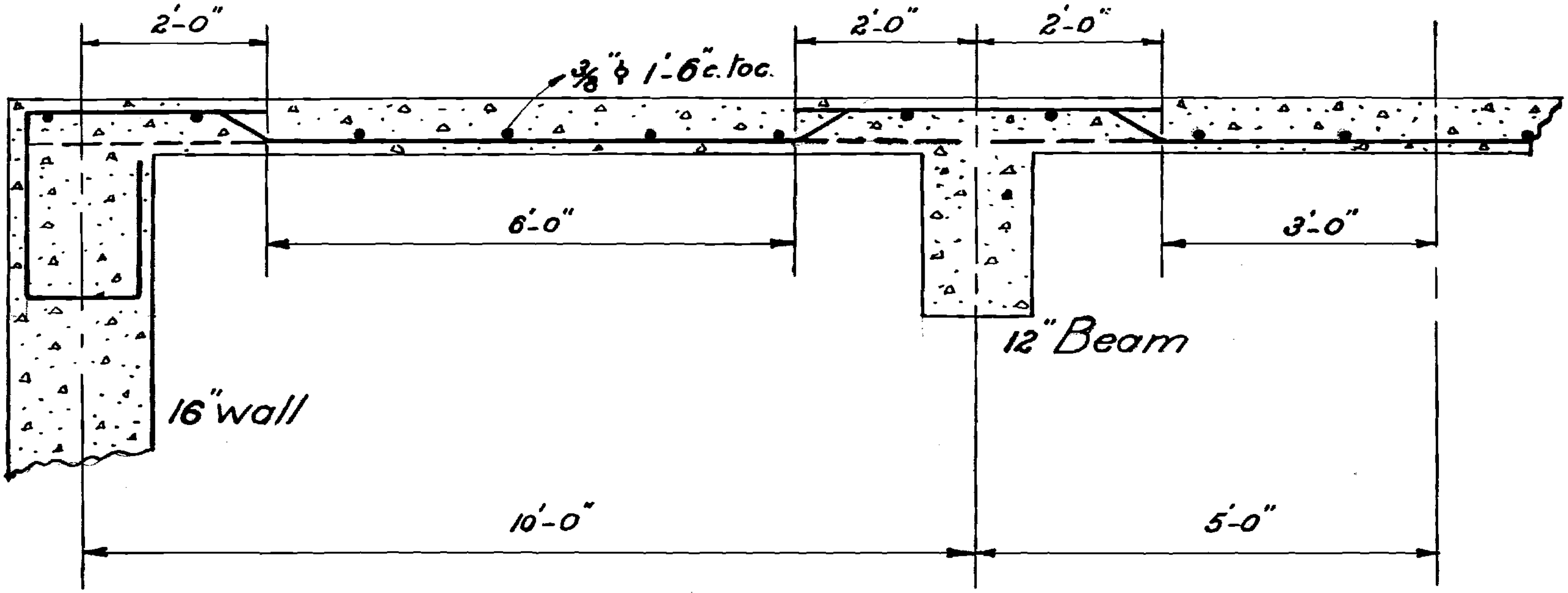
Exterior span : $A_s = 1.2 \times .0229 = .026 \text{ in.}^2/\text{in.}$

For $\frac{1}{2}$ in. rounds Allowable spacing $= \frac{.1963}{.026} = 7.55''$

USE 1/2 in. rounds at 8" c. to c.

(5 per cent variation allowable)

3/8 in. rounds at 1'- 6" c. to C. for temp. and shrinkage strs.



Depth = 5" $d = 3\frac{3}{4}$"

Neg. & Pos. steel - $\frac{1}{2}$" ϕ @ 8" c. to c.   50% bent at $\frac{1}{5}$ points
30° to the horizontal

Slab Detail Drawing

Temperature and Shrinkage steel is placed transversly to the main reinforcement.

5o per cent of the main reinforcement is bent up 30 degrees to the horizontal at 1/5 points.

Arrangement of bars is shown on the Slab detailed drawing.

Design of Floor Beams

Allowable: $f_c = 1200$ $f_s = 18000$ $n = 10$ $v = 180$ $u = 120$

$= 1350$ (at supports)

Clear span = 19'- 0"	Assume 12" Girders	Live load = 100
Bearing span = 20'- 0"		Slab 5" = 63
Panel width = 10'- 0"		Floors = 15
		Beam = ____
		Total in lbs./ft.^{2} 178

w = 180x10 + 2oo = 2000 lbs. per ft. Assuming d = 14.5''

b = 12"

$v = \frac{V}{bd}$ $V = \frac{wL}{2} = \frac{2000x19}{2} = 19000$ lbs.

$v = \frac{19000}{12x14.5} = 109$ lbs./in.^{2} being less than 180 - O.K.

$M = \frac{wL^2}{12} = \frac{2000x19^2}{12} = 66000$ ft.lbs.

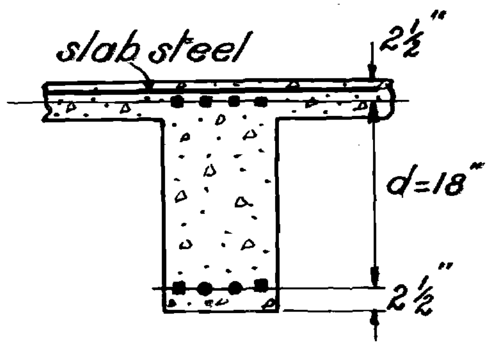
Floor Beam: Steel:

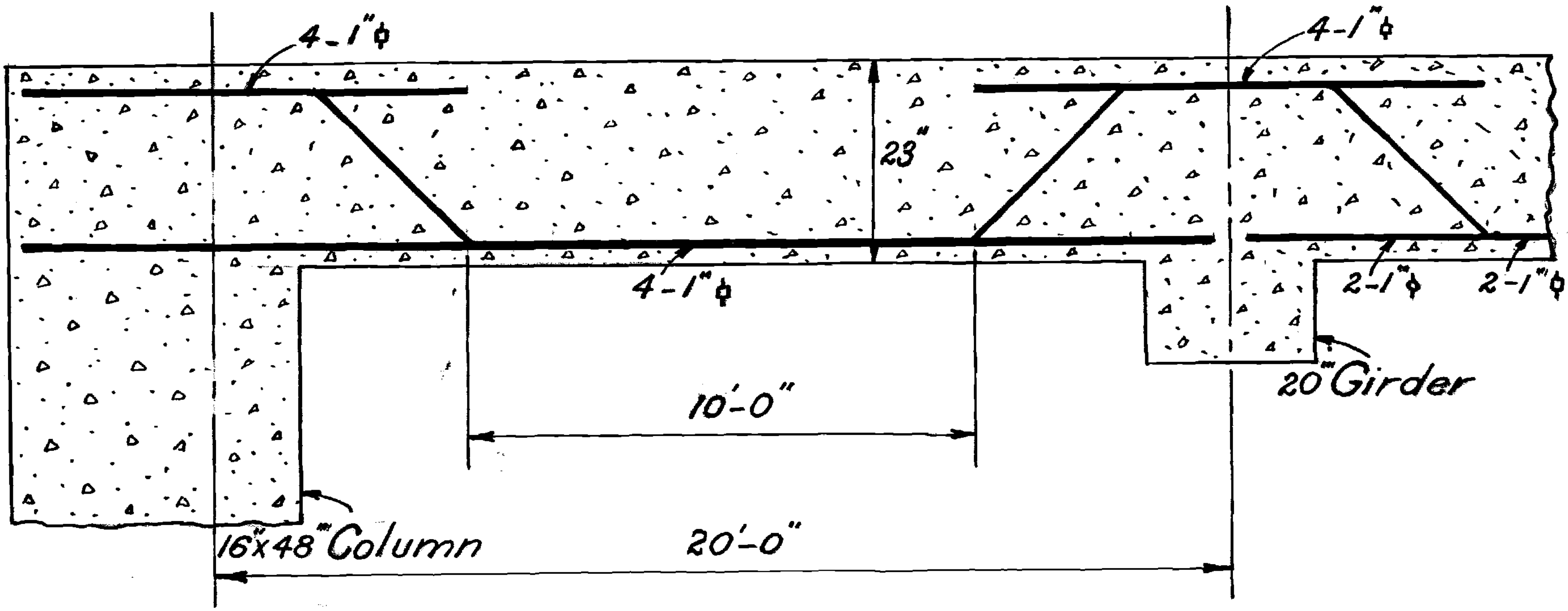
Interior span: $A_s = \frac{M}{f_sjd} = \frac{66000X12}{18000x7/8x14\ 1/2} = 3.46$ in.^{2}

USE 2 - 1 in. rounds 1.57 in.^{2}

2 - 1 in. square +2.00 in.^{2}

3.57 in.^{2} being more than 3.46 in.^{2} O.K.





Beam Detail Drawing

Exterior span: $A_s = 3.46 \times 1.2 = 4.15\ in^2$

USE 4 - 1 in. squares (Considering 5 per cent allowabe variation)

Bond :

$$u = \frac{vb}{0} = \frac{109 \times 12}{14.28} = 25\ lbs./in^2 \qquad O.K.$$

Arrangement of bars is shown on the Beam detail drawing.

Check for d :

$$d = \frac{M}{bR} = \frac{66000 \times 12}{12 \times 157} \doteq 20.5$$

USE d = 20.5"

Total thickness = 23"

Design of Floor Girders

Allowable: $f_c = 1200$ $f_s = 18000$ $n = 10$ $v = 180$ $u = 120$

= 1350 (at supports)

Clear span = 28'- 0" assume 24" columns

Bearing span = 30'- 0"

Panel width = 20'- 0"

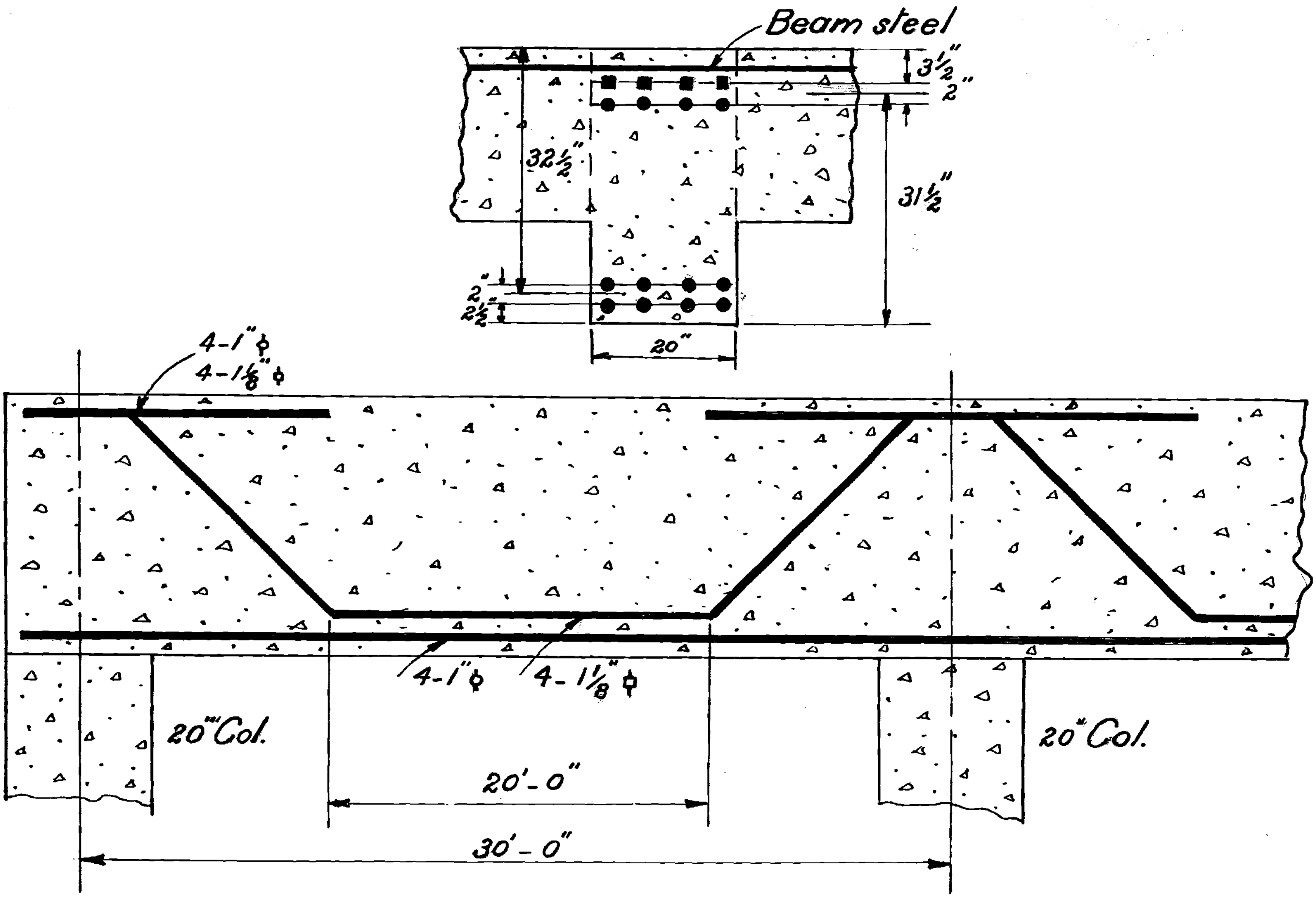
Loads : - Beam = 2 (20 x 2000) = 80000

Stem = 28 x 450 = 12600

2)92600

46300 = V

Assume: d = 33.5" b = 20"



Girder Detail Drawing

Max. $- M = \frac{L}{27}$ (6P) $= \frac{28}{27} \times 6 \times 40000 = 250000$ (due to beams)

$\frac{1}{10} \times 450 \times 28^2 = \underline{35000}$ (due to D.L. of stem)

285000 ft.lbs.

Check for: $d = \frac{285000 \times 12}{20 \times 157} = 33''$

$v = \frac{46300}{20 \times 33.5} = 66$ lbs./in.^{2} O.K.

Stem: Steel:

$A_s = \frac{285000 \times 12}{18000 \times 7/8 \times 33.5} = 8.8$ in.^{2}

USE 4 - 1 in. rounds = 3.14
4 - 1 1/8 in. sq. = 5.06 at supports.

Pos. USE 8 - 1 in. rounds = 6.28 required $A_s = \frac{8.8}{1.2} = 6.3$ in.^{2}

Check for: $u = \frac{66 \times 20}{30.57} = 43$ lbs./in.^{2} O.K.

Arrangement of bars is shown on the Girder detail drawing.

Design Procedure for Columns.

In monolithic construction where a column is part of the rigid frame, there is always a certain amount of restraint exerted at the joint and this restraint depends upon the relative stiffness of the various members, the number of spans of the beams, and the system of loadings. The determination of the bending moment in the columns is consequently more difficult than it is under simple conditions. The two general cases which need consideration in building-design are first, an interior column supporting beams of unequal spans, or beams of equal span with unbalanced live loads, and secondly, a column in an outside wall, supporting one end of a non-continious beam constructed monolithicaly with the column. In both of these cases the column is subjected to a bending moment which it must resist in addition to the direct stresses resulting from the axial loads.

Axial loads:

The total external at any crossection of any slab, beam, or girder is determined by considering all the loads and reactions on either the left or right of the section under design, and disregarding those on the other side. The maximum vertical shears, V, are immediately to the left or right of the supports. For all uniformly or symmetrically-loaded members of a continious member with fixed ends the positive or negative shears are equal to one half the the span load.

Bending moments:

Bending moments at column sections are obtained by the use of the Hardy Cross method of moment distribution. Having obtained the column section moments and axial loads in such a combination that will give maximum values the columns are designed as shown on the computation sheets.

COLUMN DESIGN COMPUTATION SHEETS

Internal columns:

M max. = 357000 ft.lbs. (from Hardy Cross diag.)

P max. = 2(46300) = 92600 (2xV in Girder)

Assumed column size = 20 x 20

Unsupported length = 10 ft.

$e = \frac{M}{P} = \frac{357000}{92600} = 3.88$ inches. (falls in the middle third)

Therefore no f_c computations required:

Steel:

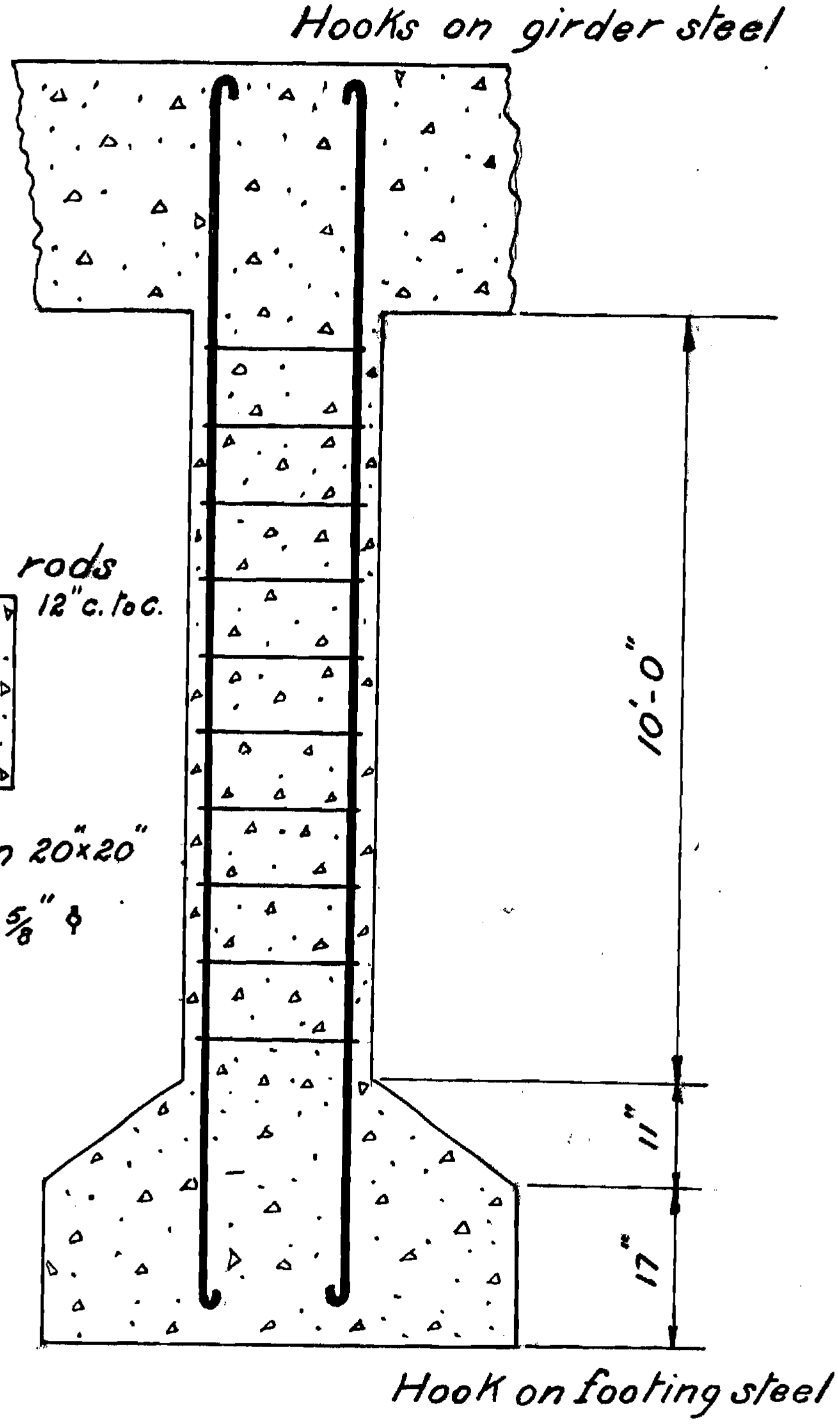
Try 4-5/8 in. rounds (minimum allowable)

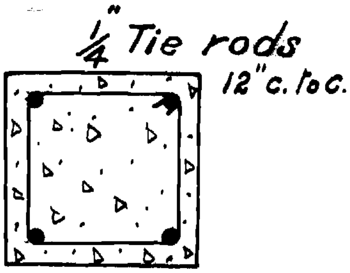
$p = \frac{As}{Ag} = \frac{1.23}{400} = .0038$

Allowable $P = .225 x f'_c \times A_g \left[1 + (n-1)p\right]$

$= .225x3000x400 \quad 1-(9).0038 = 278000$ lbs. O.K.

USE values assumed above with 1/4 in. tie rods at 12" c.toc.





Crossection 20"x20"
Steel - 4 - 5/8" ϕ

Internal Column Details

External columns. (Along the short side of the building)

$$M_{max.} = 437000 \text{ ft.lbs.}$$

$$P_{max.} = V \text{ in girder} + D.L. \text{ of wall}$$

$$= 46300 + 9000 \text{ (at center, h of wall, 40')}$$

USE same size columns as Internal ones. Though the moment value is greater the column is laterally supported (partly) on the outside, and using the same size column as the Internal ones is not unsafe.

External columns. (Along the longer sides of the building)

As there is no moment and horizontal reaction at the springing of the rib, the axial load on the external columns consists of the 30 lbs./ft.^{2} Live Load on the horizontal projection of the roof, the Dead Load of the rib (obtained from column 6 in the Roof Frame design), and the weight of the slab, forming the roof, supported by it. Plus the beam reaction.

Slab:

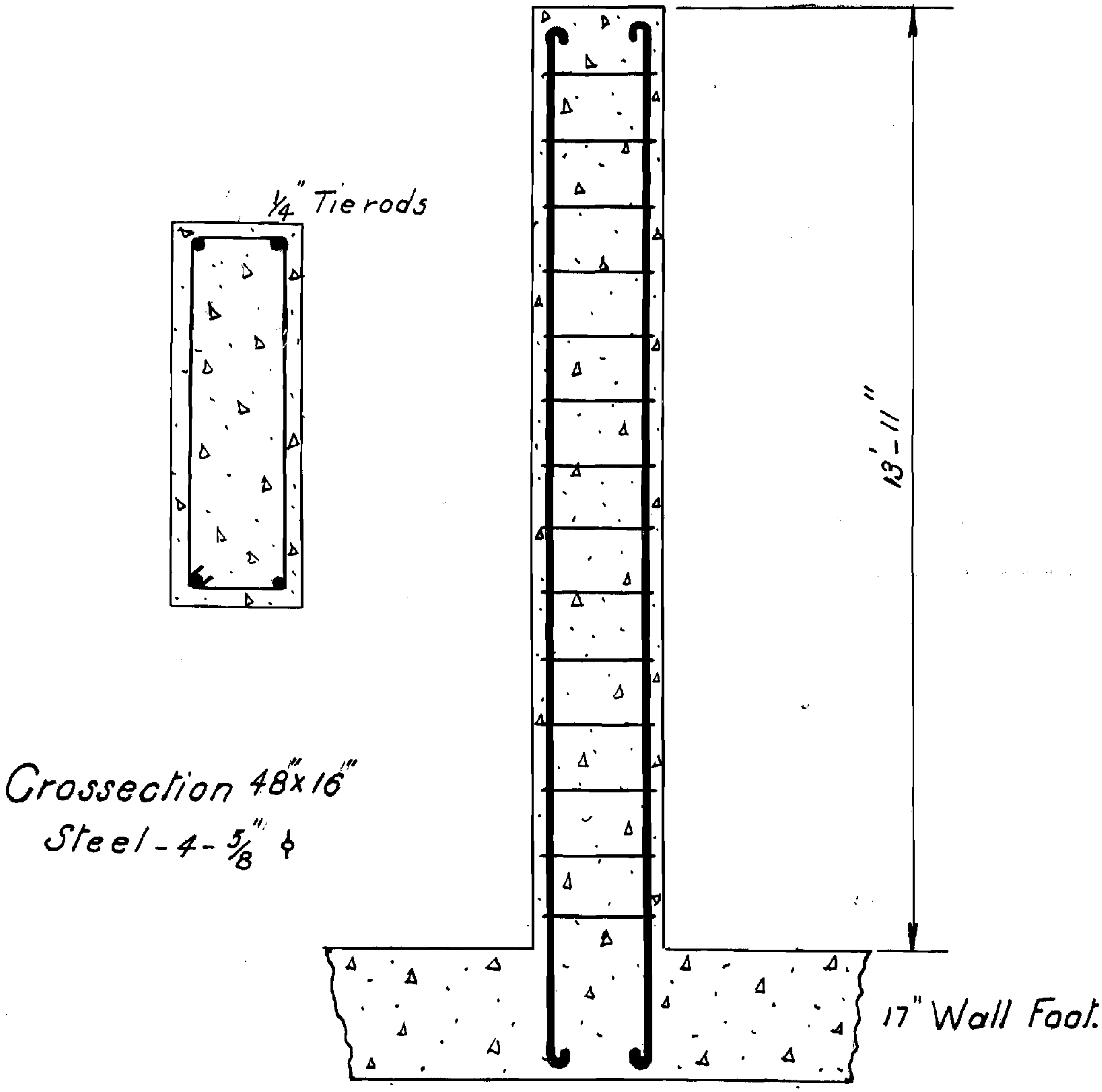
$$\text{Area} = bd$$

$$b = \frac{2\pi r}{4} = 20\pi$$

$$d = 10'$$

$$\text{Volume} = 100\pi = 314 \text{ ft}^3$$

		Slab weight	= 47000 lbs.
Rib weight:			= 18650 "
Beam reaction:			= 19000 "
Live load:	30xArea of hor. proj. = 30 x 400		= 12000 "
		Load on Column	= 96650 lbs.



External Column Details

The size of the column should be so that the rib rests on it firm - ly, that is - should have the same dimensions that the rib has at the springing.

Therefore:

Column size = 48" x 16"

This column being much larger than the required size, no design is necessary, and the amout of steel used is the minimum allowable.

Steel : 4 - 5/8" rounds.

1/4 in. Tie rods 12 in. c. to c.

<u>Corner columns.</u>

Use same size columns as the internal ones.

BASEMENT WALLS.

When the basement of a building is located below grade it is necessary to design the exterior walls to resist earth pressure. A minimum width of 12 inches will be considered in our case. The figure on the next page shows the load diagram, and details of design, for a basement wall subjected to earth pressure.

Let:

P = total pressure per linear foot against the back of the wall, in pounds;

h = the height of the wall, that is span length in ft.

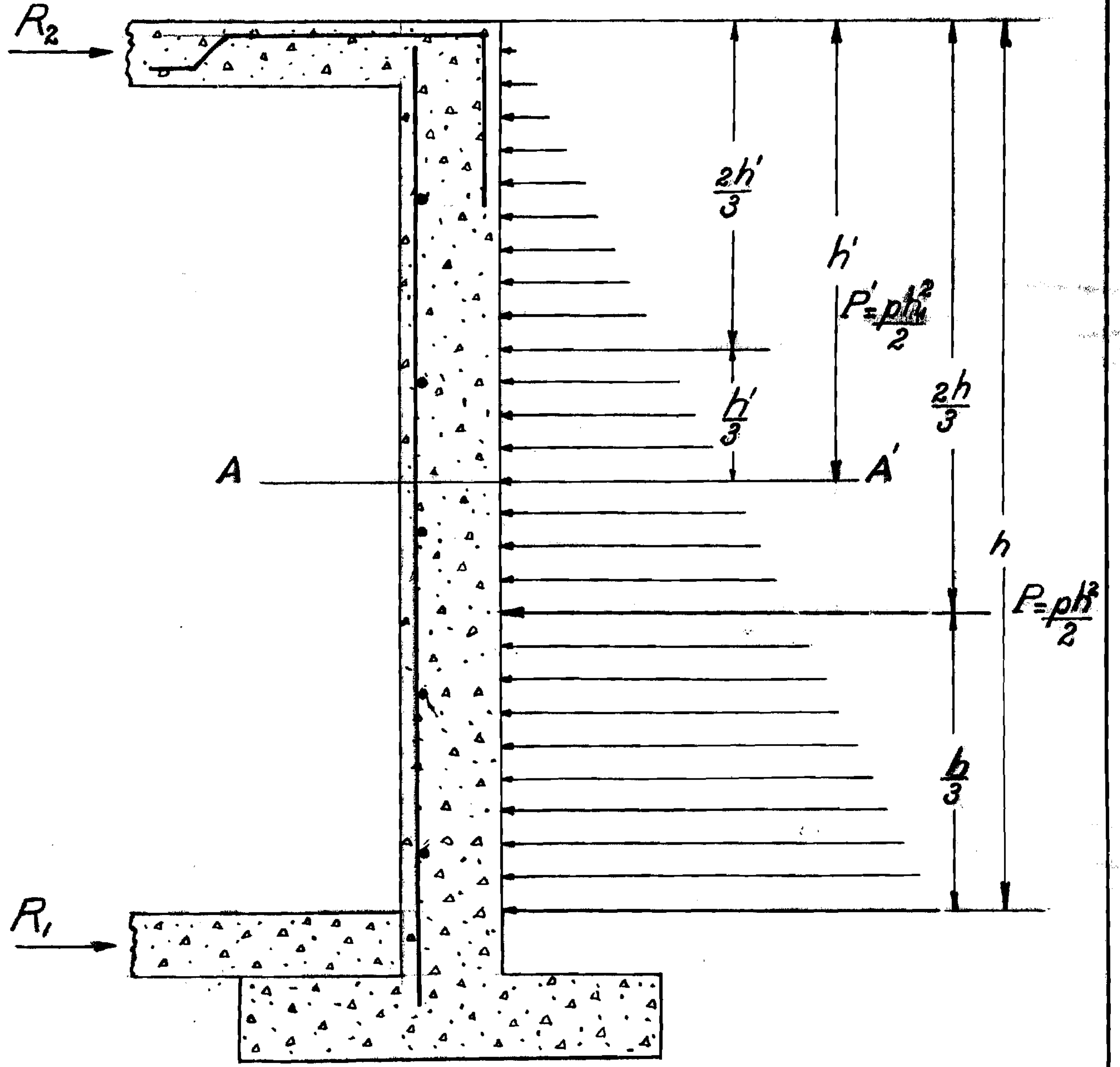
p = equivalent fluid pressure of the soil in lbs./in.^{2}

The total pressure $P = \frac{ph^2}{2}$ and the resultant acts at a height $\frac{h}{3}$ above the bottom of the wall, the reactions per linear ft. of wall are then, $R_1 = \frac{2P}{3}$ and $R_2 = \frac{P}{3}$. Considering any section, such as A - A', it is apparent that the total pressure P', on the part of above this section equals $\frac{ph_1^2}{2}$ and its resultant acts at a distance $\frac{h_1}{3}$ above the section.

Taking the momemts about the section A - A' :

$$M_{A-A'} = R_2 h_1 - \frac{P' h_1}{3}$$

Remembering that $\frac{P}{P'} = \frac{h^2}{h_1^2}$ and substituting the values of $R_2 = \frac{P}{3}$ and $P' = \frac{Ph_1^2}{h^2}$



Basement Wall

$$M_{A-A'} = \frac{Ph_1}{3} - \frac{Ph_1^2}{h^2}x:\frac{h_1}{3}$$

Or,

$$M_{A-A'} = \frac{Ph_1}{3}\left(1- \frac{h_1^2}{h^2}\right)$$

This is the expression for the bending moment at any section of the wall.

The bending moment is a maximum when the section A-A' is that of zero shear or, since the shear at any section is equal to the sum of reactions minus the sum of the loads to the right or left of the section, when $R_2 - P' = 0$.

Transposing the values of:

$$R_2 = \frac{P}{3} = \frac{Ph^2}{6} \quad \text{and} \quad P' = \frac{ph_1^2}{2},$$

$$h_1^2 = \frac{h^2}{3},$$

and the depth of the section of the max. bending moment is

$$h_1 = .58\ h$$

The value of the max. moment is

$$M = R_2(0.58h) - P'\frac{.58\ h}{3},$$

Again making the same substitutions,

$$M = \frac{ph^2}{6} \times 0.58h - \frac{p(.058)^3}{6}$$

which gives a value of

$$M = 0.064ph^3 \text{ ft.lbs.}$$

The weight of the soil may be assumed to be 100lbs./ft.^{3}, and to exert a pressure equal to that of a fluid weighing 30 lbs./ft.^{3}

Therefore $p = 30$.

Computations for Side Walls.

<u>No surcharge</u>:

$$h = 13 \text{ ft.} \qquad p = 30 \text{ lbs./in}^2$$

$$P = \frac{169 \times 30}{2} = 2540 \text{ lbs. per linear ft.}$$

Section of max. M is at,

0.58 x 13 = 7.5 ft. from the top of the wall

Max. bending moment,

$$M = .064 \times 30 \times 2197 = 4220 \text{ ft.lbs/linear ft.}$$

Max, horizontal shear,

$$V = R_1 = 2/3\ P = 2/3 \times 2540 = 1700 \text{ lbs.}$$

d = 10'' (Having assumed a thickness of 12 inches.)

$$A_s = \frac{4220 \times 12}{18000 \times 7/8 \times 10} = .324 \text{in}^2/1 \text{ ft. width}$$

Try 3/8 in. rounds at 16 in. c. to c.

$$A_s = .44 \times \frac{12}{16} = .33 \text{ in}^2 \quad \text{O.K.}$$

$$u = \frac{1700}{1.178 \times 7/8 \times 10} \times \frac{16}{12} = 220 \qquad v = \frac{1700}{12 \times 10 \times 7/8} = 16.2 \quad \text{O.K.}$$

<u>USE</u> : 3/8 in. rounds at 8 in. c, to c. (for bond)

Distributing bars - 3/8 in. rounds at 18 in. c. to c.
(shrinkage and temperature)

Walls along the short sides of the building are 16 in. thick.

<u>USE</u> : 3/8 in. rounds at 12 in. c. to c. and same distributing bars.

FOOTINGS.

Loads on footings. The Load used in determining the Area of a Footing is the load used in the design of the column in the storey immediately above the footing considered, plus the weight of the footing itself. The load from a basement floor resting on the material of the footing bed need not be considered as an added weight on the footing.

The DESIGN-LOAD, used in determining the thickness and reinforcement of the footing slab, is the load as defined above, less the weight of the footing. For a footing resting on the foundation bed this load, is assumed to be uniformly distributed over the area of the footing.

Bending Moments in Footings. The general equation for the maximum bending moment occuring at the supports in a uniformly loaded cantilever is $M = WL/2$ in which W represents the total distributed load and L the length of the projection.

$$M = 6wl^2 \text{ in in.lbs.}$$

The moment found by the above formula, occurs at the face of the column. The depth of the concrete determined by the maximum moment is :

$$d = \sqrt{\frac{M}{Rb}}$$ b taken as 12 inches.

In our design the depth of the footing is determined by the use of the punching shear, which is computed for the whole or a part of the column section periphery, considering the portion of the total shear that is tributary to the section chosen.

Diagonal tension in Footings. As in the case of beam design, the sheaeing stress developed in a section is considered to be a measure of the diagonal tension. Since the vertical shear at any section of a beam is equal to the sum of the reactions minus the sum of the loads to the left or right of the section, V is determined, in the case of footings, by considering the soil reactions as positive and the column loads as negative, their algebraic sum being the vertical shear at any section. Since it is difficult and expensive to use diagonal tension reinforcement in footings the diagonal tension shear is kept low.

Bond stresses in Footings. Bond stresses usually are very high in footings. When straight rods are used for reinforcement, without hooks at the ends for anchorage, bond usually controls the selection of steel. The shear for bond computations is that at the face of wall or column.

Layout of Footings.

On account of varying soil conditions and the presence of different levels in basement floors due to special constructions such as pits for elevators or machinery, it is not common for all footings in a building to have their bases at the same elevation. It is not convenient to have a variety of lengths for the basement columns and the necessity for this can be avoided by using plain concrete pedestals on all footings extending to the underside of the basement floor. In the design of our building the use of pedestals is not required.

COMPUTATIONS FOR INTERIOR COLUMN FOOTING

Footing: Interior Column -

Loads:

column wt. = 41700 lbs. (size, 20x20, height, 10')

P = 92600 lbs. (value used in column design)

Total =134300 lbs.

Stresses:

$f_s = 18000$ $v = 60$ $u = 90$

$n = 10$ $j = 7/8$ Soil value = 8000 lbs./□'

Depth:

Soil Reaction under column -

$$\text{approx.} = \frac{20 \times 20}{144} \times 8000 = 22260 \text{ lbs.}$$

Total punching shear -

$$= 134300 - 22260 = 112100 \text{ lbs.}$$

$H = .6(H+h)$ $h = .4(H+h)$ L = length of base (square)

$$H + h = \frac{112100}{60 \times 80} + 4 = 27.4 \text{ in.} \qquad \text{USE } 28 \text{ in.}$$

Therefore: $H = .6 \times 28 = 16.8$ in. Use 17 in.

$h = .4 \times 28 = 11.2$ in. Use 11 in.

Size:

Weight of footing: (approx.)

Footing of same volume and uniform thickness has total thickness

$$= H + \frac{\text{volume of cap}}{L^2}$$

$$= 0.6(H + h) + \frac{(.4L)^2 \left[(.4)(H + h) \right]}{L^2}$$

$$= 0.664 \ (H + h)$$

Weight of base,

$$= .664 \ (H + h) \ \frac{150}{12}$$

$$= 232 \text{ lbs./ft.}^2$$

Net pressure alowable on footing,

$$= 8000 - 232 = 7768 \text{ lbs./ft.}^2$$

$$L^2 = \frac{134300}{7768} = 17.3 \qquad L = 4'\text{-}2''$$

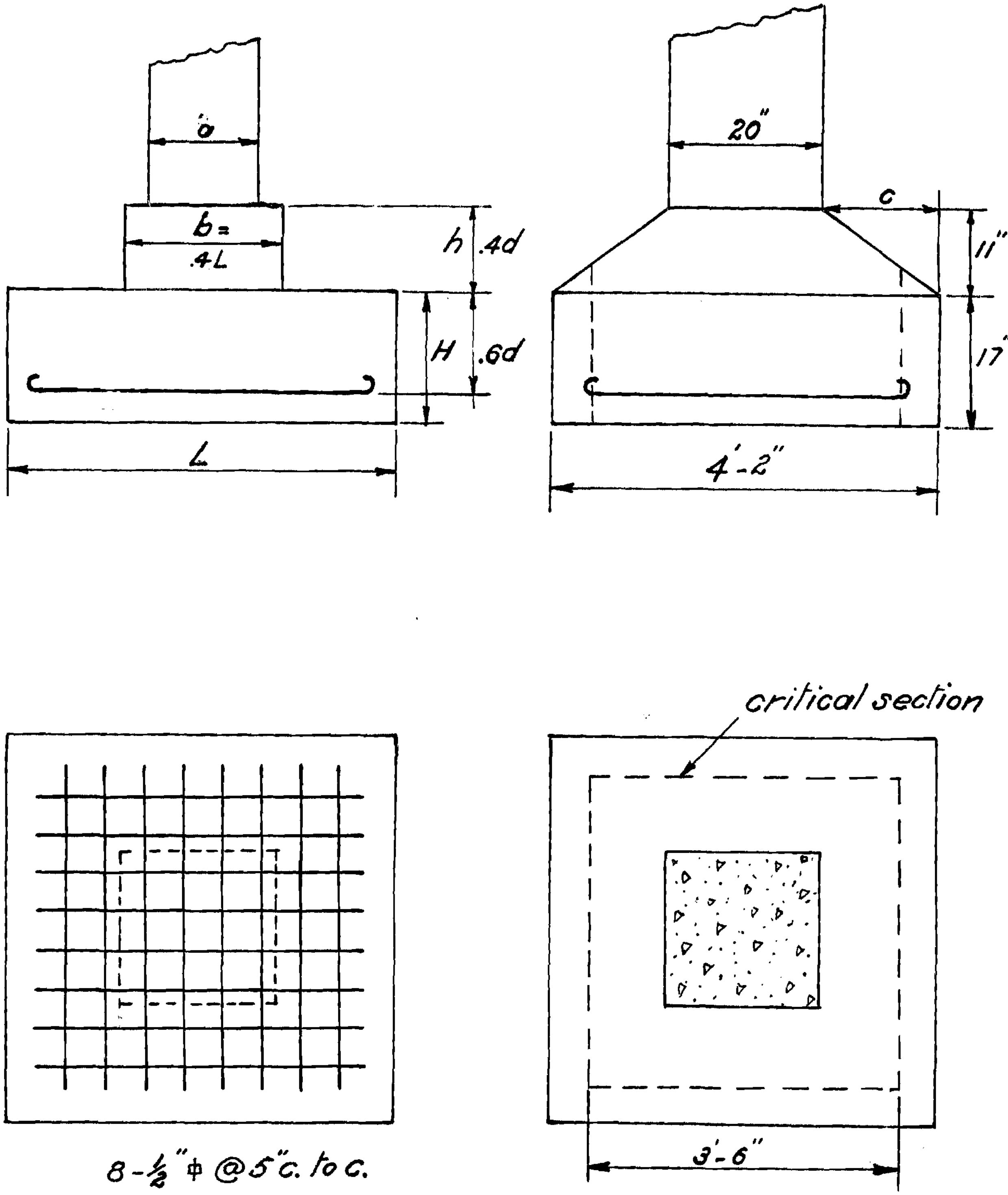
$b = .4 \times 50 = 20''$ (Use no cap, use sloped top)

Diagonal tension:

$$\text{Net pressure} = \frac{134300}{(4.17)^2} = 7700 \text{ lbs./ft.}^2$$

Total shear on critical section,

$$= 7700 \left[(4.17)^2 - (3.93)^2 \right] = 92500 \text{ lbs.}$$



Detailed Drawings of Int.
Footings.

Unit shear $= \frac{92500}{4x40x7/8x15} = 44$ lbs./in.^{2} O.K.

Steel:

$M = \frac{W}{2}(a + 1.2\ c)\ c^2$ $a = 20''$ $c = \frac{30}{2} = 15$ in.

$= \frac{7700}{2}(\frac{20}{12} + 1.2x1.25)(1.25)^2$

$= 19000$ ft.lbs.

$A_s = \frac{19000 \times 12}{18000 \times 7/8 \times 24} = .605$ in.^{2}

Check for bond:

In this case the bond value governs the steel.

$\Sigma O = \frac{V}{7/8x24x90}$ $u = 90$ lbs./in.^{2} allowable

$V = 1/4x7700[(4.17)^2 - (1.66)^2] = 28200$ lbs.

$\Sigma O = \frac{28200}{7/8x24x90} = 14.9$ in.

USE 8 - 1/2 in. square bars. $\Sigma O = 16$ " O.K.

Use of footings may or may not be necessary under the external columns, as wall footings are to be used.

Final dimentions and arrangement of bars are shown on the detailed footing drawings.

Footing: Exterior column (along the short side of the building).

USE same size footings as used for the interior column footings.

ALSO USE same size footings at the four corners of the building.

Footing: Exterior column (along the long side of the building)

Column size 48" x 16"

Loads:

Axial load = 96650 lbs.

Column weight $= 13 \times \frac{48x16}{144} \times 150 = 10400$ lbs.

Total Load = 107050 lbs.

Soil reaction under column = 5.34 x 8000 = 42700 lbs.

Trying to have the column sit on the wall footing, we see that:

Net pressure using a (5 ft^2) footing -

$$\frac{107050}{25} = 4300 \text{ lbs./ft}^2 \quad \text{O.K.}$$

Shear at critical section $= \frac{107050}{168x14} = 45.5$ O.K.

The values above show that having the columns sit on the wall footings would be safe.

Wall Footings.

Wall Footing: (along the short side of the building)

Load:

Two short ends of the building are composed of a semi circularly shaped 16'' end wall, and a 16" basement wall, the weight of these two walls is supported by the wall footing. As the load varies along the length of the wall footing an average load is used in designing the wall footing, for convenience.

Max. Load:

$$53 \times 1.33 \times 150 = 95000 \text{ lbs./linear ft.}$$

Height of end wall (at center) = 40 ft.

Height of basement wall = 13 ft.

Min. Load:

$$13 \times 1.33 \times 150 = 26000 \text{ lbs./linear ft.}$$

Use 60000 lbs./linear ft. including the weight of the footing itself.

Soil value = 8000 lbs./ft.^{2} Footing width $= \frac{60000}{8000} = 7.50'$

Therefore the projection on each side of the wall is 3'(1)

The design load is the net unit soil pressure, exclusive of the weight of the footing.

$$w = \frac{60000 - 1000}{7.5} = 7860 \text{lbs./ft.}^2$$

Maximum bending momemt:

$$M = 6wl^2 = 6 \times 7860 \times 9 = 425000 \text{ in.lbs.}$$

Depth governed by moment:

$$d = \sqrt{\frac{425000}{12x157}} = 15''$$

Check for shear:

$$v = \frac{(1 - \frac{d}{12})\ w}{10.5\ x\ d} = \frac{(3 - 1.25)7860}{10.5\ x\ 15} = 87\ lbs./in.^2$$

Allowable v = 90 lbs./in.^{2} (Using special anch.)

As the value H, at the periphery of the exterior column (along the short side of the building) is 17", and a protective covering of 3" is necessary, using a d of 14" would be very convenient. The diffence caused on the moment and shear values, by making this change is negligible.

Steel:

$$A_s = \frac{425000}{18000x7/8x14} = 1.93\ in^2$$ Try 1 in. rounds

Number of bars per ft. $= \frac{1.93}{.78} = 2.48$

Spacing $= \frac{12}{2.48} = 4.85''$ Use $4\frac{1}{2}''$

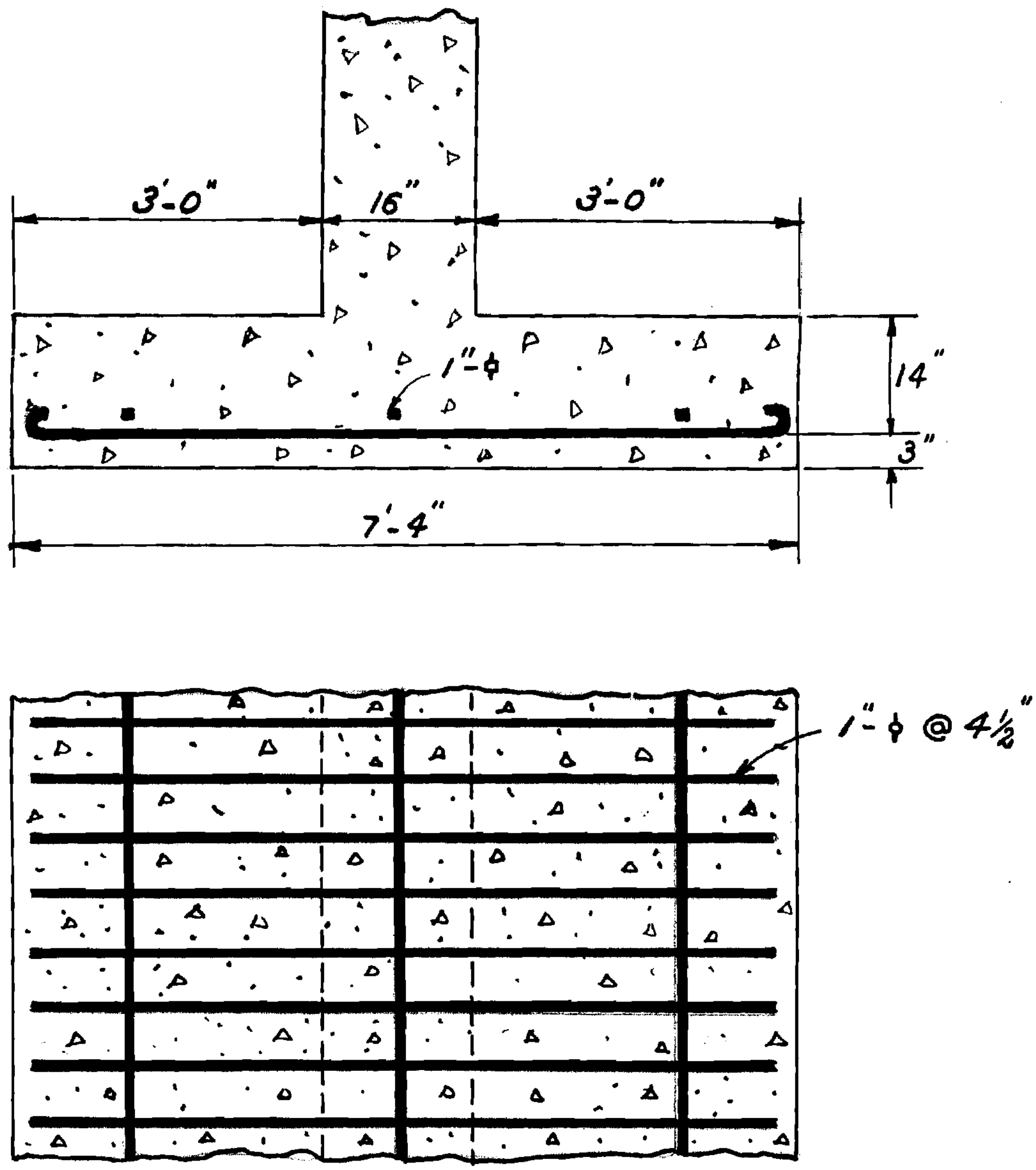
Check for bond:

$$u = \frac{7860\ x\ 3}{2.48x3.142x14} = 189\ lbs./in^2$$

Allowable u = 180 lbs./in^2 (Using special anchorage)

USE --- 1 in. rounds at $4\frac{1}{2}''$ c. to c.

Arrangement of bars are shown on the Wall Footing Detail drawings.



Wall Footing (short side of building)

Wall Footing: (along long side of building)

Load:

The load to be used for the design of the wall footing depth consists of the weight of the basement wall plus the reaction from floor beams.

Beam reaction = 19000 lbs.

Weight of wall = 13 x 1 x 150 = 1950 lbs.

USE 4000 lbs./linear ft.

Soil value.

$$8000 \text{ lbs./ft}^2$$

Footing width:

$$\frac{40000}{8000} = 5 \text{ ft.}$$

Therefore the projection on each side of the wall is 2' (1). Assuming the weight of the footing to be 1000 lbs./ft^2

$$w = \frac{40000 - 1000}{5} = 7800 \text{ lbs./ft}^2$$

Maximum bending moment:

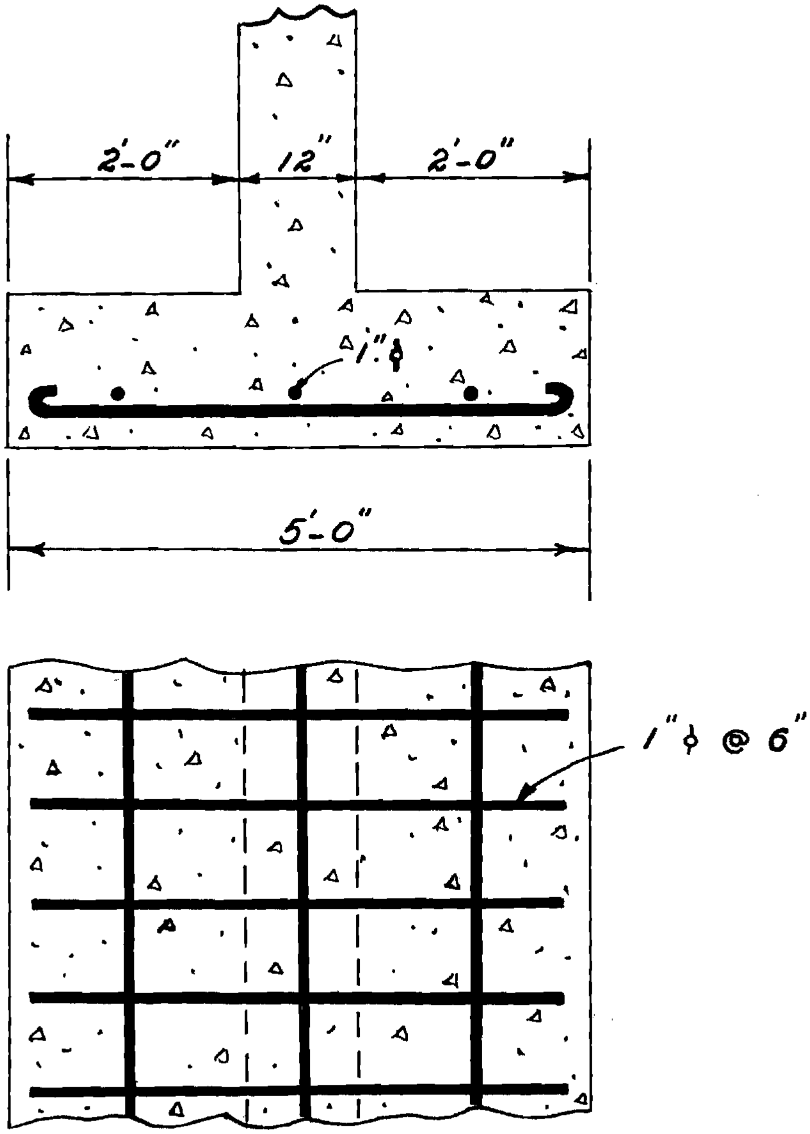
$$M = 6 \times 7800 \times 4 = 187000 \text{ in.lbs.}$$

So that all the footings will be of the same level , a total depth of 17" with a d of 14" would be of convenience. This selection makes it unnecessary to check for the shear in the footing.

Steel:

$$A_s = \frac{187000}{18000 \times 7/8 \times 14} = .85 \text{ in}^2 \qquad \text{Try 1" rounds.}$$

$$\text{Number of bars per ft.} = \frac{.85}{.78} = 1.1$$



Wall Footing (long side of building)

A spacing of lo.5 ft. for the bars could be used according to the steel requirements, but the effect of bond must be considered before final decision.

Allowable u = 180 lbs./in^2 (special anchorage)

Number of bars per ft. $= \frac{7800 \times 2}{180 \times 3.142 \times 14} = 1.97$

Max. spacing $= \frac{12}{1.97} = 6.1"$

USE 1 in. rounds at 6" c. to c.

Basement Floor.

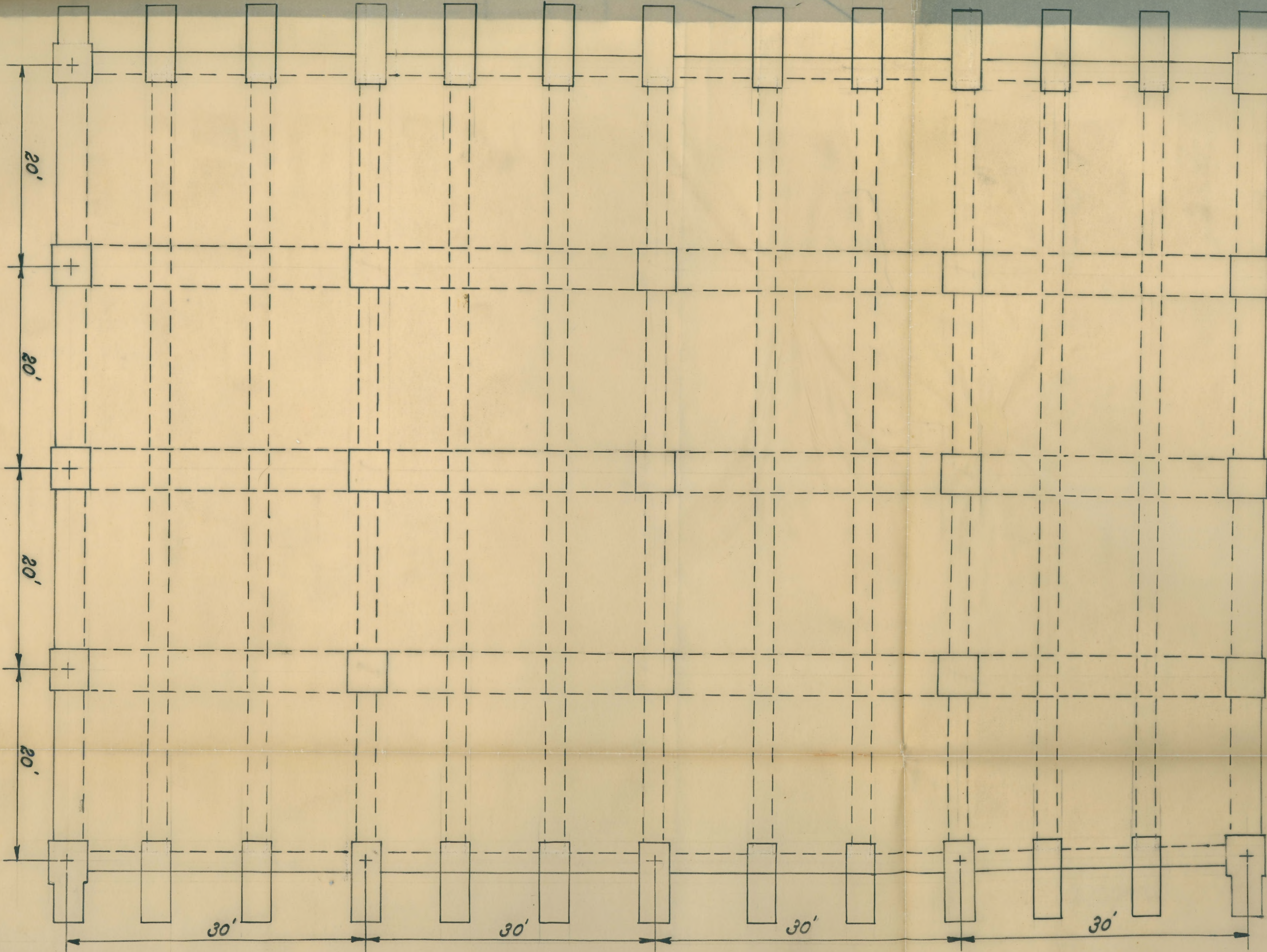
The basement floor consits of a 6" slab having the necessary reinforcement for temperature and shrinkage, and proper drainage openings where the showers and lavatories are located.

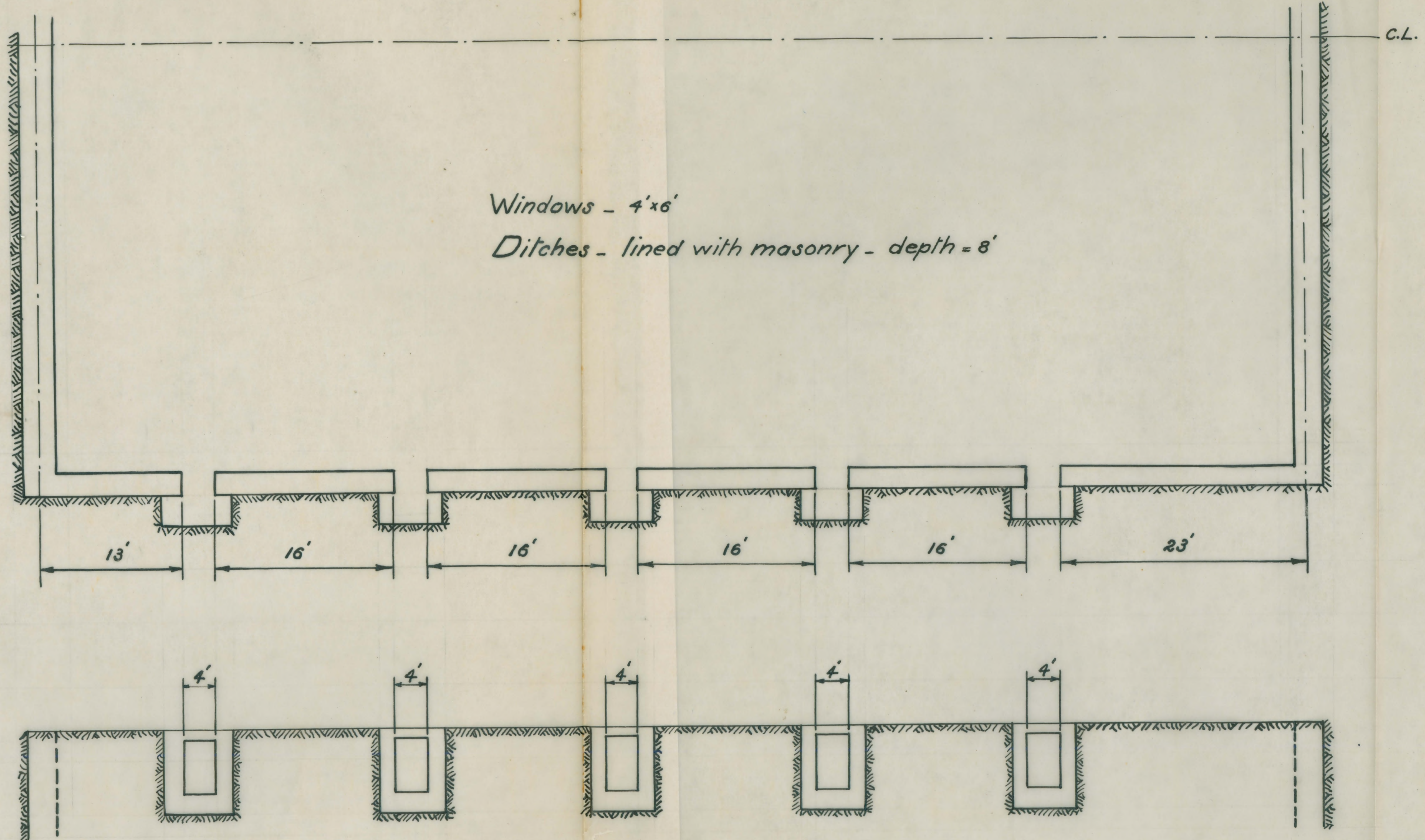
Windows.

The windows, at the basement are located as shown on the drawing drawn through the horizontal section of the basement walls. Windows along the roof frame may or may not be spaced like the ones at the basement.

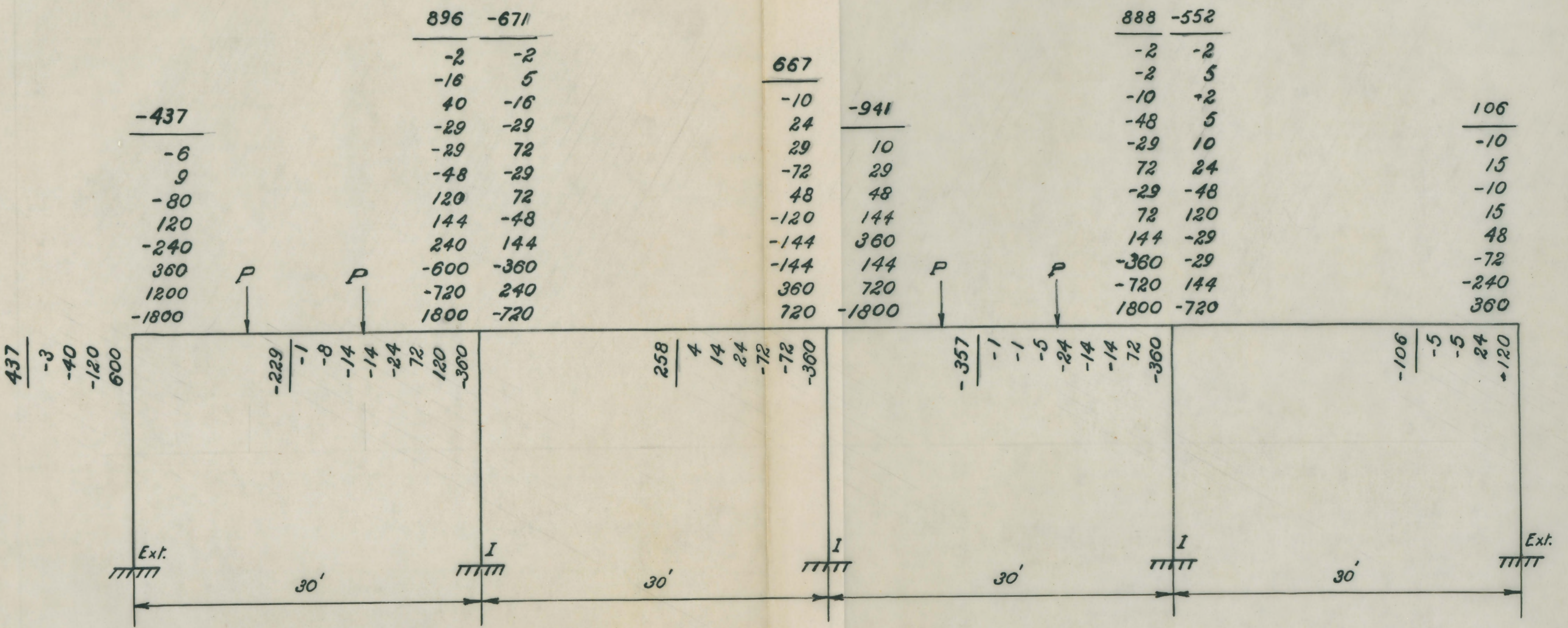
THE END.

First Floor Plan





Section Thru Basement Walls - Windows

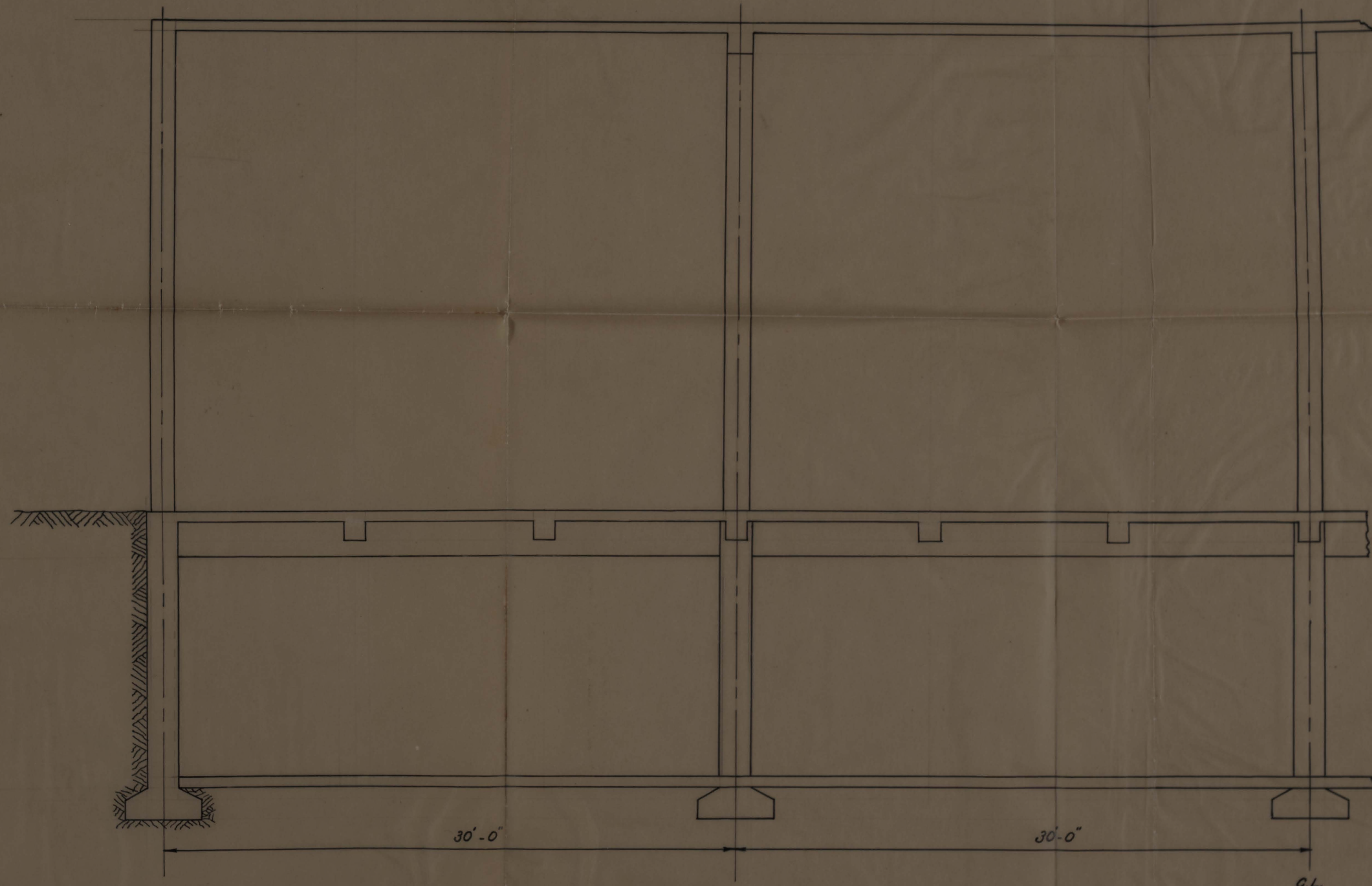


Moments in ft. Kips

K column -(1)

K girder -(2)

Column, Girder Mom. Distribution



SECTION THRU BUILDING